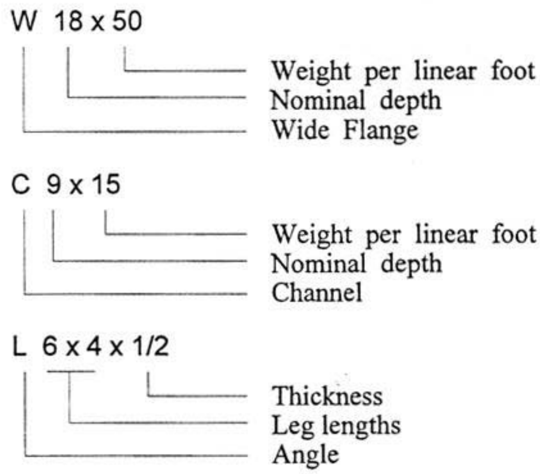
Steel Design

Notation:

a = name for width dimension
A = name for area
A_g = gross area, equal to the total area ignoring any holes
$A_{req'd\text{-}adj}$ = area required at allowable stress when shear is adjusted to include self weight
A_w = area of the web of a wide flange section, as is A_{web}
AISC = American Institute of Steel Construction
ASD = allowable stress design
b = name for a (base) width
= name for height dimension
b_f = width of the flange of a steel beam cross section
B = width of a column base plate
B_1 = factor for determining M_u for combined bending and compression
c = largest distance from the neutral axis to the top or bottom edge of a beam. as is c_{max}
c_1 = coefficient for shear stress for a rectangular bar in torsion
C_b = modification factor for moment in ASD & LRFD steel beam design
C_m = modification factor accounting for combined stress in steel design
C_v = web shear coefficient
d = name for depth
= depth of a wide flange section
D = shorthand for dead load
DL = shorthand for dead load
E = shorthand for earthquake load
= modulus of elasticity
f_a = axial stress
f_b = bending stress
f_p = bearing stress
f_v = shear stress
$f_{v\text{-}max}$ = maximum shear stress
f_y = yield stress
F = shorthand for fluid load
F_a = allowable axial (compressive) stress
F_b = allowable bending stress
F_{cr} = flexural buckling stress
F_e = elastic critical buckling stress
F_p = allowable bearing stress
F_u = ultimate stress prior to failure
F_y = yield strength
F_{yw} = yield strength of web material
h = name for a height
h_c = height of the web of a wide flange steel section
H = shorthand for lateral pressure load
I = moment of inertia with respect to neutral axis bending
I_y = moment of inertia about the y axis
J = polar moment of inertia
k = distance from outer face of W flange to the web toe of fillet
= shape factor for plastic design of steel beams
K = effective length factor for columns, as is k
l = name for length, as is L
= column base plate design variable
L = name for length or span length, as is l
= shorthand for live load
L_b = unbraced length of a steel beam in LRFD design
L_e = effective length that can buckle for column design, as is ℓ_e
L_r = shorthand for live roof load
= maximum unbraced length of a steel beam in LRFD design for inelastic lateral-torsional buckling
L_p = maximum unbraced length of a steel beam in LRFD design for full plastic flexural strength
LL = shorthand for live load
LRFD = load and resistance factor design
m = edge distance for a column base plate
M = internal bending moment
M_a = required bending moment (ASD)
M_{max} = maximum internal bending moment
$M_{max\text{-}adj}$ = maximum bending moment adjusted to include self weight

M_n = nominal flexure strength with the full section at the yield stress for LRFD beam design
M_p = internal bending moment when all fibers in a cross section reach the yield stress
M_u = maximum moment from factored loads for LRFD beam design
M_y = internal bending moment when the extreme fibers in a cross section reach the yield stress
n = edge distance for a column base plate
n' = column base plate design value
$n.a.$ = shorthand for neutral axis
N = bearing length on a wide flange steel section
= depth of a column base plate
P = name for load or axial force vector
P_a = required axial force (ASD)
P_c = available axial strength
P_{e1} = Euler buckling strength
P_r = required axial force
P_n = nominal column load capacity in LRFD steel design
P_p = nominal bearing capacity of concrete under base plate
P_u = factored column load calculated from load factors in LRFD steel design
r = radius of gyration
R = generic load quantity (force, shear, moment, etc.) for LRFD design
= shorthand for rain or ice load
R_a = required strength (ASD)
R_n = nominal value (capacity) to be multiplied by ϕ in LRFD and divided by the safety factor Ω in ASD
R_u = factored design value for LRFD design
S = shorthand for snow load
= section modulus
$S_{req'd}$ = section modulus required at allowable stress
$S_{req'd\text{-}adj}$ = section modulus required at allowable stress when moment is adjusted to include self weight
t_f = thickness of flange of wide flange
t_{min} = minimum thickness of column base plate
t_w = thickness of web of wide flange
T = torque (axial moment)
= shorthand for thermal load
V = internal shear force
V_a = required shear (ASD)
V_{max} = maximum internal shear force
$V_{max\text{-}adj}$ = maximum internal shear force adjusted to include self weight
V_n = nominal shear strength capacity for LRFD beam design
V_u = maximum shear from factored loads for LRFD beam design
$w_{equivalent}$ = the equivalent distributed load derived from the maximum bending moment
$w_{self\ wt}$ = name for distributed load from self weight of member
W = shorthand for wind load
X = column base plate design value
Z = plastic section modulus of a steel beam
Δ_{actual} = actual beam deflection
$\Delta_{allowable}$ = allowable beam deflection
Δ_{limit} = allowable beam deflection limit
Δ_{max} = maximum beam deflection
ε_y = yield strain (no units)
ϕ = resistance factor
ϕ_b = resistance factor for bending for LRFD
ϕ_c = resistance factor for compression for LRFD
ϕ_v = resistance factor for shear for LRFD
λ = column base plate design value
γ = load factor in LRFD design
π = pi (3.1415 radians or 180°)
ρ = radial distance
Ω = safety factor for ASD

Steel Design



Structural design standards for steel are established by the *Manual of Steel Construction* published by the American Institute of Steel Construction, and uses **Allowable Stress Design** and **Load and Factor Resistance Design**. The 13th edition combines both methods in one volume and provides common requirements for analyses and design and requires the application of the same set of specifications.

Materials

American Society for Testing Materials (ASTM) is the organization responsible for material and other standards related to manufacturing. Materials meeting their standards are guaranteed to have the published strength and material properties for a designation.

A36 – carbon steel used for plates, angles	F_y = 36 ksi, F_u = 58 ksi, E = 29,000 ksi
A572 – high strength low-alloy used for some beams	F_y = 60 ksi, F_u = 75 ksi, E = 30,000 ksi
A992 – for building framing used for most beams	F_y = 50 ksi, F_u = 65 ksi, E = 30,000 ksi

(A572 Grade 60 has the same properties as A992)

ASD $$R_a \le R_n/\Omega$$

where R_a = required strength (dead or live; force, moment or stress)
R_n = nominal strength specified for ASD
Ω = safety factor

Factors of Safety are applied to the limit stresses for allowable stress values:

bending (braced, $L_b < L_p$)	Ω = 1.67
bending (unbraced, $L_p < L_b$ and $L_b > L_r$)	Ω = 1.67 (nominal moment reduces)
shear (beams)	Ω = 1.67
shear (bolts)	Ω = 2.00 (tabular nominal strength)
shear (welds)	Ω = 2.00

- L_b is the unbraced length between bracing points, laterally
- L_p is the limiting laterally unbraced length for the limit state of yielding
- L_r is the limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling

LRFD $R_u \leq \phi R_n$ $where \cdots R_u = \Sigma \gamma_i R_i$

where ϕ = resistance factor
γ = load factor for the type of load
R = load (dead or live; force, moment or stress)
R_u = factored load (moment or stress)
R_n = nominal load (ultimate capacity; force, moment or stress)

Nominal strength is defined as the

> capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths (such as yield strength, F_y, or ultimate strength, F_u) and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions

Factored Load Combinations

The design strength, ϕR_n, of each structural element or structural assembly must equal or exceed the design strength based on the ASCE-7 combinations of factored nominal loads:

$1.4(D + F)$
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
$1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2D + 1.0E + L + 0.2S$
$0.9D + 1.6W + 1.6\text{ H}$
$0.9D + 1.0E + 1.6\text{ H}$

Criteria for Design of Beams

Allowable normal stress or normal stress from LRFD should not be exceeded:

$$F_b \text{ or } \phi F_n \geq f_b = \frac{Mc}{I}$$
$$(M_a \leq M_n / \Omega \text{ or } M_u \leq \phi_b M_n)$$

Knowing M and F_b, the minimum section modulus fitting the limit is:

$$Z_{req'd} \geq \frac{M_a}{F_y \Omega} \quad \left(S_{req'd} \geq \frac{M}{F_b} \right)$$

Besides strength, we also need to be concerned about *serviceability*. This involves things like limiting deflections & cracking, controlling noise and vibrations, preventing excessive settlements of foundations and durability. When we know about a beam section and its material, we can determine beam deformations.

Determining Maximum Bending Moment

Drawing V and M diagrams will show us the maximum values for design. Computer applications are very helpful.

Determining Maximum Bending Stress

For a prismatic member (constant cross section), the maximum normal stress will occur at the maximum moment.

For a *non-prismatic* member, the stress varies with the cross section AND the moment.

Deflections

Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well.

Elastic curve equations can be superpositioned ONLY if the stresses are in the elastic range. *The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*

Allowable Deflection Limits

All building codes and design codes limit deflection for beam types and damage that could happen based on service condition and severity.

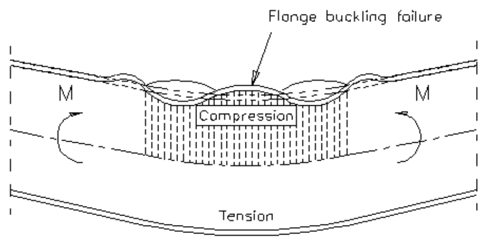
$$\Delta_{actual} \leq \Delta_{allowable} = L/_{value}$$

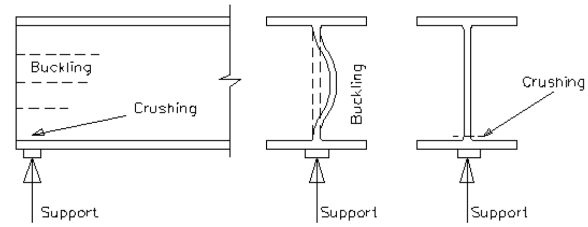
Use	LL only	DL+LL
Roof beams:		
Industrial	L/180	L/120
Commercial		
plaster ceiling	L/240	L/180
no plaster	L/360	L/240
Floor beams:		
Ordinary Usage	L/360	L/240
Roof or floor (damageable elements)		L/480

Lateral Buckling

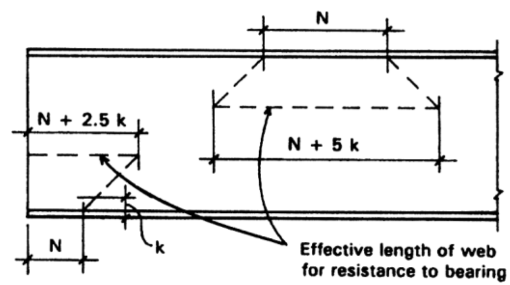
With compression stresses in the top of a beam, a sudden “popping” or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger I_y.

Local Buckling in Steel I Beams– Web Crippling or Flange Buckling





Concentrated forces on a steel beam can cause the web to buckle (called web crippling). Web stiffeners under the beam loads and bearing plates at the supports reduce that tendency. Web stiffeners also prevent the web from shearing in plate girders.



The maximum support load and interior load can be determined from:

$$P_{n\,(\max-\text{end})} = (2.5k + N)F_{yw}t_w$$

$$P_{n\,(\text{interior})} = (5k + N)F_{yw}t_w$$

where t_w = thickness of the web
N = bearing length
k = dimension to fillet found in beam section tables

ϕ = 1.00 (LRFD) Ω = 1.50 (ASD)

Beam Loads & Load Tracing

In order to determine the loads on a beam (or girder, joist, column, frame, foundation...) we can start at the top of a structure and determine the *tributary area* that a load acts over and the beam needs to support. Loads come from material weights, people, and the environment. This area is assumed to be from half the distance to the next beam over to halfway to the next beam.

The reactions must be supported by the next lower structural element *ad infinitum*, to the ground.

LRFD Bending or Flexure

For determining the flexural design strength, $\phi_b M_n$, for resistance to pure bending (no axial load) in most flexural members where the following conditions exist, a single calculation will suffice:

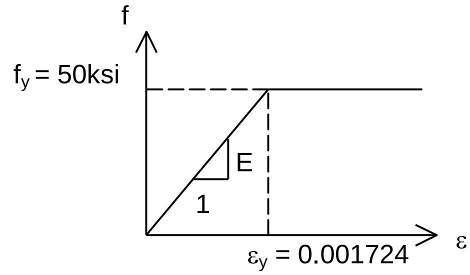
$$\Sigma\gamma_i R_i = M_u \le \phi_b M_n = 0.9F_y Z$$

where M_u = maximum moment from factored loads
ϕ_b = resistance factor for bending = 0.9

M_n = nominal moment (ultimate capacity)
F_y = yield strength of the steel
Z = plastic section modulus

Plastic Section Modulus

Plastic behavior is characterized by a yield point and an increase in strain with no increase in stress.



Internal Moments and Plastic Hinges

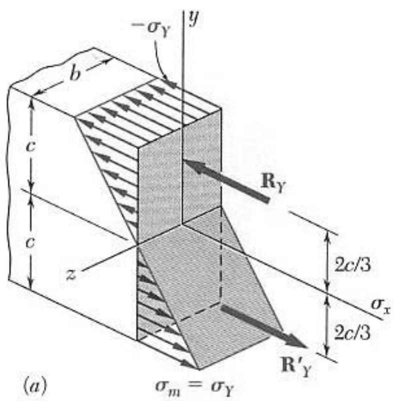
Plastic hinges can develop when all of the material in a cross section sees the yield stress. Because all the material at that section can strain without any additional load, the member segments on either side of the hinge can rotate, possibly causing instability.

For a rectangular section:

Elastic to f_y:

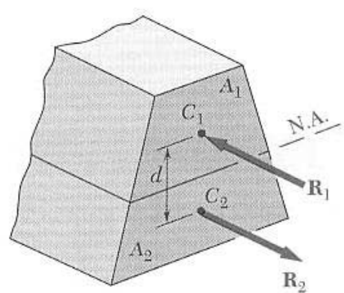
$$M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y$$

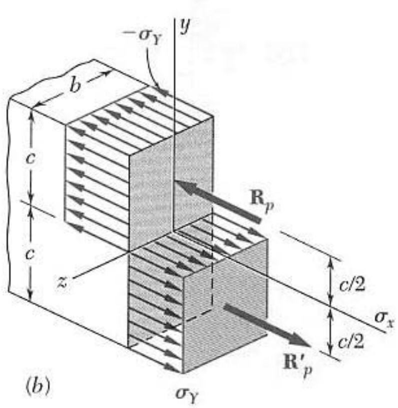
Fully Plastic: $$M_{ult} \text{ or } M_p = bc^2 f_y = \tfrac{3}{2} M_y$$

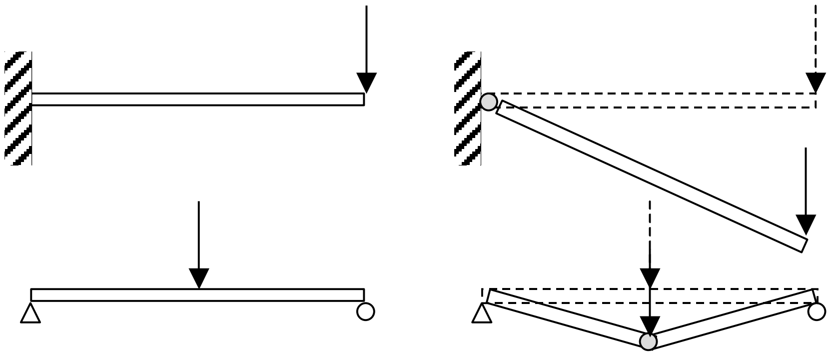


For a non-rectangular section and internal equilibrium at σ_y, the n.a. *will not necessarily be at the centroid.* The n.a. occurs where the $A_{tension} = A_{compression}$. The reactions occur at the centroids of the tension and compression areas.

$$A_{tension} = A_{compression}$$







Instability from Plastic Hinges

Shape Factor:

The ratio of the plastic moment to the elastic moment at yield:

$$k = M_p / M_y$$

k = 3/2 for a rectangle
k ≈ 1.1 for an I beam

Plastic Section Modulus

$$Z = \frac{M_p}{f_y} \quad \text{and} \quad k = Z/S$$

Design for Shear

$$V_a \le V_n / \Omega \text{ or } V_u \le \phi_v V_n$$

The nominal shear strength is dependent on the cross section shape. Case 1: With a thick or stiff web, the shear stress is resisted by the web of a wide flange shape (with the exception of a handful of W's). Case 2: When the web is not stiff for doubly symmetric shapes, singly symmetric shapes (like channels) (excluding round high strength steel shapes), inelastic web buckling occurs. When the web is very slender, elastic web buckling occurs, reducing the capacity even more:

1. For $h/t_w \le 2.24\sqrt{\frac{E}{F_y}}$ $V_n = 0.6F_{yw}A_w$ ϕ_v = 1.00 (LRFD) Ω = 1.50 (ASD)

   where *h* equals the clear distance between flanges less the fillet or corner radius for rolled shapes
   V_n = nominal shear strength
   F_{yw} = yield strength of the steel in the web
   A_w = $t_w d$ = area of the web

2. For $h/t_w > 2.24\sqrt{\frac{E}{F_y}}$ $V_n = 0.6F_{yw}A_wC_v$ ϕ_v = 0.9 (LRFD) Ω = 1.67 (ASD)

   where C_v is a reduction factor (1.0 or less by equation)

Design for Flexure

$M_a \leq M_n / \Omega$ or $M_u \leq \phi_b M_n$ ϕ_b = 0.90 (LRFD) Ω = 1.67 (ASD)

The nominal flexural strength M_n is the *lowest* value obtained according to the limit states of

1. yielding, limited at length $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$, where r_y is the radius of gyration in *y*
2. lateral-torsional buckling limited at length L_r
3. flange local buckling
4. web local buckling

Beam design charts show available moment, M_n/Ω and $\phi_b M_n$, for unbraced length, L_b, of the compression flange in one-foot increments from 1 to 50 ft. for values of the bending coefficient C_b = 1. For values of $1<C_b\leq 2.3$, the required flexural strength M_u can be reduced by dividing it by C_b. (C_b = 1 when the bending moment at any point within an unbraced length is larger than that at both ends of the length. C_b of 1 is conservative and permitted to be used in any case. When the free end is unbraced in a cantilever or overhang, C_b = 1. The full formula is provided below.)

NOTE: the self weight <u>is not</u> included in determination of $\phi_b M_n$

<u>Compact Sections</u>

For a laterally braced *compact* section (one for which the plastic moment can be reached before local buckling) only the limit state of yielding is applicable. For unbraced compact beams and non-compact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable.

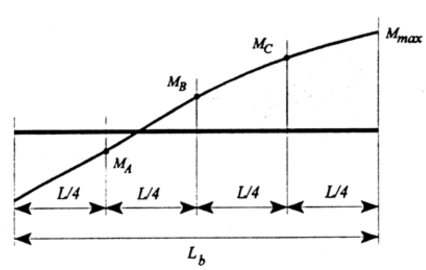
Compact sections meet the following criteria: $\frac{b_f}{2t_f} \leq 0.38\sqrt{\frac{E}{F_y}}$ and $\frac{h_c}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$

where:

b_f = flange width in inches
t_f = flange thickness in inches
E = modulus of elasticity in ksi
F_y = minimum yield stress in ksi
h_c = height of the web in inches
t_w = web thickness in inches

With lateral-torsional buckling the nominal flexural strength is

$$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$



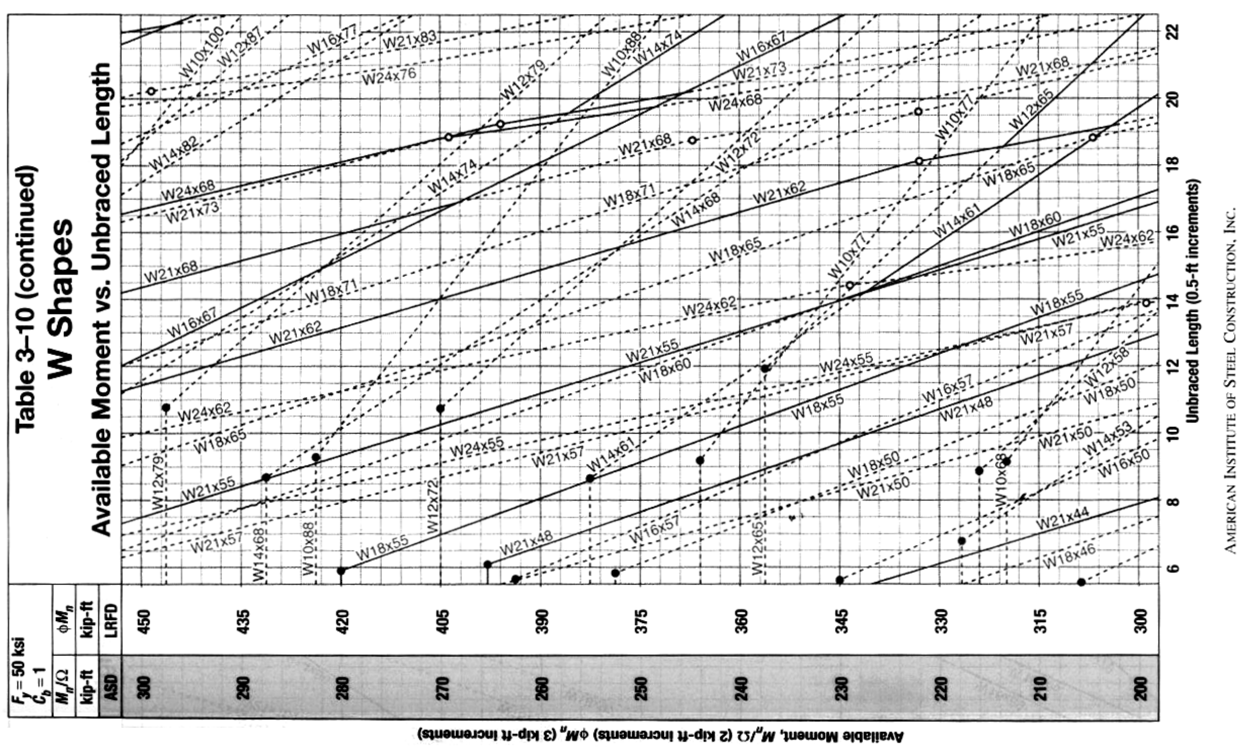
where C_b is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

M_{max} = absolute value of the maximum moment in the unbraced beam segment
M_A = absolute value of the moment at the quarter point of the unbraced beam segment
M_B = absolute value of the moment at the center point of the unbraced beam segment
M_C = absolute value of the moment at the three quarter point of the unbraced beam segment length.

Available Flexural Strength Plots

Plots of the available moment for the unbraced length for wide flange sections are useful to find sections to satisfy the design criteria of $M_a \le M_n/\Omega$ *or* $M_u \le \phi_b M_n$. The maximum moment that can be applied on a beam (taking self weight into account), M_a or M_u, can be plotted against the unbraced length, L_b. The limit L_p is indicated by a solid dot (•), while L_r is indicated by an open dot (○). Solid lines indicate the most economical, while dashed lines indicate there is a lighter section that could be used. C_b, which is a modification factor for non-zero moments at the ends, is 1 for simply supported beams (0 moments at the ends). (see *figure)*



Design Procedure

The intent is to find the most light weight member (which is economical) satisfying the section modulus size.

1. Determine the unbraced length to choose the limit state (yielding, lateral torsional buckling or more extreme) and the factor of safety and limiting moments. Determine the material.
2. Draw V & M, finding V_{max} and M_{max}.for unfactored loads (ASD, V_a & M_a) or from factored loads (LRFD, V_u & M_u)
3. Calculate $S_{req'd}$ *or* Z when yielding is the limit state. This step is equivalent to determining if $f_b = \frac{M_{max}}{S} \leq F_b$, $S_{req'd} \geq \frac{M_{max}}{F_b} = \frac{M_{max}}{F_y/\Omega}$ and $Z \geq \frac{M_u}{\phi_b F_b}$ to meet the design criteria that

   $$M_a \leq M_n/\Omega \text{ or } M_u \leq \phi_b M_n$$

   If the limit state *is something other than yielding*, determine the nominal moment, M_n, or use plots of available moment to unbraced length, L_b.

4. For steel: use the section charts to find a trial S *or Z and remember that the beam self weight (the second number in the section designation) will increase $S_{req'd.}$* or *Z* The design charts show the lightest section within a grouping of similar S's or Z's.

TABLE 9.1 Load Factor Resistance Design Selection

		F_y = 36 ksi			
Designation	Z_x in.^{3}	L_p ft	L_r ft	M_p kip-ft	M_r kip-ft
W 33 × 141	**514**	**10.1**	**30.1**	**1,542**	**971**
W 30 × 148	500	9.50	30.6	1,500	945
W 24 × 162	468	12.7	45.2	1,404	897
W 24 × 146	418	12.5	42.0	1,254	804
W 33 × 118	**415**	**9.67**	**27.8**	**1,245**	**778**
W 30 × 124	408	9.29	28.2	1,224	769
W 21 × 147	373	12.3	46.4	1,119	713
W 24 × 131	370	12.4	39.3	1,110	713
W 18 × 158	356	11.4	56.5	1,068	672

*****Determine the "updated" V_{max} and M_{max} including the beam self weight, and verify that the updated $S_{req'd}$ has been met.*******

5. Evaluate horizontal shear using V_{max}. This step is equivalent to determining if $f_v \leq F_v$ is satisfied to meet the design criteria that $V_a \leq V_n/\Omega$ or $V_u \leq \phi_v V_n$

   For I beams: $$f_{v-max} = \frac{3V}{2A} \approx \frac{V}{A_{web}} = \frac{V}{t_w d} \qquad V_n = 0.6F_{yw}A_w \quad or\ V_n = 0.6F_{yw}A_wC_v$$

   Others: $$f_{v-max} = \frac{VQ}{Ib}$$

6. Provide adequate bearing area at supports. This step is equivalent to determining if $f_p = \frac{P}{A} \leq F_p$ is satisfied to meet the design criteria that $P_a \leq P_n/\Omega$ or $P_u \leq \phi P_n$
7. Evaluate shear due to torsion $f_v = \frac{T\rho}{J}$ *or* $\frac{T}{c_1 ab^2} \leq F_v$ (circular section or rectangular)
8. Evaluate the deflection to determine if $\Delta_{max LL} \leq \Delta_{LL-allowed}$ and/or $\Delta_{max\,Total} \leq \Delta_{Tota-lallowed}$

***** note: when $\Delta_{calculated} > \Delta_{limit}$, $I_{required}$ can be found with: and $S_{req'd}$ will be satisfied for similar self weight ******

$$I_{req'd} \geq \frac{\Delta_{too\,big}}{\Delta_{limit}} I_{trial}$$

FOR ANY EVALUATION:

Redesign (with a new section) at any point that a stress or serviceability criteria is NOT satisfied and re-evaluate each condition until it is satisfactory.

Load Tables for Uniformly Loaded Joists & Beams

Tables exist for the common loading situation of uniformly distributed load. The tables either provide the safe distributed load based on bending and deflection limits, they give the allowable span for specific live and dead loads including live load deflection limits.
If the load is *not uniform*, an *equivalent uniform load* can be calculated from the maximum moment equation:

$$M_{max} = \frac{w_{equivalent}L^2}{8}$$

If the deflection limit is less, the design live load to check against allowable must be increased, ex.

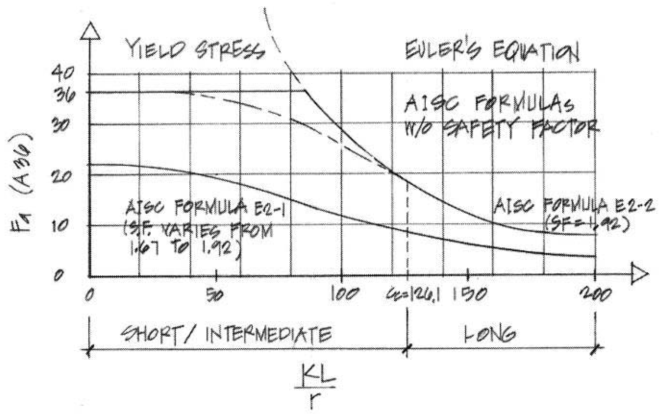
$$w_{adjusted} = w_{ll-have}\left(\frac{L/360}{L/400}\right) \begin{matrix} \textit{table limit} \\ \textit{wanted} \end{matrix}$$

Criteria for Design of Columns

If we know the loads, we can select a section that is adequate for strength & buckling.

If we know the length, we can find the limiting load satisfying strength & buckling.

Design for Compression



American Institute of Steel Construction (AISC) Manual 13th ed:

$P_a \le P_n / \Omega$ or $P_u \le \phi_c P_n$ where

$P_u = \Sigma \gamma_i P_i$

γ is a load factor
P is a load type
φ is a resistance factor
P_n is the nominal load capacity (strength)

φ = 0.90 (LRFD) Ω = 1.67 (ASD)

For compression $P_n = F_{cr} A_g$

where : A_g is the cross section area and F_{cr} is the flexural buckling stress

The flexural buckling stress, F_{cr}, is determined as follows:

when $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ or ($F_e \ge 0.44F_y$):

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] F_y$$

when $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ or ($F_e < 0.44F_y$):

$$F_{cr} = 0.877F_e$$

where F_e is the elastic critical buckling stress: $$F_e = \frac{\pi^2 E}{\left(KL/r\right)^2}$$

Design Aids

Tables exist for the value of the flexural buckling stress based on slenderness ratio. In addition, tables are provided in the AISC Manual for Available Strength in Axial Compression based on the effective length with respect to least radius of gyration, r_y. If the critical effective length is about the largest radius of gyration, r_x, it can be turned into an effective length about the y axis with the fraction r_x/r_y.

Sample AISC Table for Available Strength in Axial

F_y = 50 ksi

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

Shape	W12×									
Wt/ft	96		87		79		72		65	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length KL (ft) with respect to least radius of gyration r_y										
0	844	1270	766	1150	694	1040	633	951	571	859
6	811	1220	735	1100	667	1000	607	913	548	824
7	800	1200	725	1090	657	987	598	899	540	811
8	787	1180	713	1070	646	971	588	884	531	798
9	772	1160	699	1050	634	952	577	867	520	782
10	756	1140	685	1030	620	932	565	849	509	765
11	739	1110	669	1010	606	910	551	828	497	747
12	720	1080	652	980	590	887	537	807	484	727
13	701	1050	634	953	573	862	522	784	470	706
14	680	1020	615	924	556	836	506	761	456	685
15	659	990	595	895	538	809	490	736	441	662
16	637	957	575	864	520	781	473	710	425	639
17	614	923	554	833	501	752	455	684	409	615
18	591	888	533	801	481	723	437	657	393	591
19	567	852	511	769	461	694	419	630	377	566
20	543	816	490	736	442	664	401	603	360	541
22	495	744	446	670	402	603	365	548	327	491
24	447	672	402	605	362	544	328	493	294	442
26	401	602	360	541	323	486	293	440	262	393
28	356	534	319	479	286	430	259	389	231	347
30	312	469	279	420	250	376	226	340	202	303
32	274	412	246	369	220	331	199	299	177	267
34	243	365	218	327	195	293	176	265	157	236
36	217	326	194	292	174	261	157	236	140	211
38	195	292	174	262	156	234	141	212	126	189
40	176	264	157	236	141	212	127	191	114	171
Properties										
P_{wo} (kips)	137	206	121	181	104	157	90.9	136	78.2	117
P_{wi} (kips/in.)	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5
P_{wb} (kips)	296	445	243	366	185	278	142	213	106	159
P_{fb} (kips)	152	228	123	185	101	152	84.0	126	68.5	103
L_p (ft)	10.9		10.8		10.8		10.7		11.9	
L_r (ft)	46.6		43.0		39.9		37.4		35.1	
A_g (in.^{2})	28.2		25.6		23.2		21.1		19.1	
I_x (in.^{4})	833		740		662		597		533	
I_y (in.^{4})	270		241		216		195		174	
r_y (in.)	3.09		3.07		3.05		3.04		3.02	
Ratio r_x/r_y	1.76		1.75		1.75		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	23800		21200		18900		17100		15300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

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Procedure for Analysis

1. Calculate KL/r for each axis (if necessary). The largest will govern the buckling load.
2. Find F_{cr} as a function of KL/r from the appropriate equation (above) or table.
3. Compute $P_n = F_{cr} \cdot A_g$

   or alternatively compute $f_c = P_a/A$ or P_u/A
4. Is the design satisfactory?

   Is $P_a \leq P_n/\Omega$ or $P_u \leq \phi_c P_n$? $\Rightarrow$ yes, it is; no, it is no good

   or Is $f_c \leq F_{cr}/\Omega$ or $\phi_c F_{cr}$? $\Rightarrow$ yes, it is; no, it is no good

Procedure for Design

1. Guess a size by picking a section.
2. Calculate KL/r for each axis (if necessary). The largest will govern the buckling load.
3. Find F_{cr} as a function of KL/r from appropriate equation (above) or table.
4. Compute $P_n = F_{cr} \cdot A_g$

   or alternatively compute $f_c = P_a/A$ or P_u/A
5. Is the design satisfactory?

   Is $P \leq P_n/\Omega$ or $P_u \leq \phi_c P_n$? yes, it is; *no, pick a bigger section and go back to step 2.*

   Is $f_c \leq F_{cr}/\Omega$ or $\phi_c F_{cr}$? $\Rightarrow$ yes, it is; *no, pick a bigger section and go back to step 2.*
6. Check design efficiency by calculating percentage of stress used:=

$$\frac{P_a}{P_n/\Omega} \cdot 100\% \; or \; \frac{P_u}{\phi_c P_n} \cdot 100\%$$

   If value is between 90-100%, it is efficient.

   If values is less than 90%, *pick a smaller section and go back to step 2.*

Columns with Bending (Beam-Columns)

In order to *design* an adequate section for allowable stress, we have to start somewhere:

1. Make assumptions about the limiting stress from:
   - buckling
   - axial stress
   - combined stress

1. See if we can find values for r or A or Z (S for ASD)

2. Pick a trial section based on if we think r or A is going to govern the section size.

3. Analyze the stresses and compare to allowable using the allowable stress method or interaction formula for eccentric columns.

4. Did the section pass the stress test?
   - If not, do you *increase* r or A or S?
   - If so, is the difference really big so that you could *decrease* r or A or S to make it more efficient (economical)?

5. Change the section choice and go back to step 4. Repeat until the section meets the stress criteria.

Design for Combined Compression and Flexure:

The interaction of compression and bending are included in the form for two conditions based on the size of the required axial force to the available axial strength. This is notated as P_r (either P from ASD or P_u from LRFD) for the axial force being supported, and P_c (either P_n/Ω for ASD or $\phi_c P_n$ for LRFD). The increased bending moment due to the P-Δ effect must be determined and used as the moment to resist.

For $\frac{P_r}{P_c} \geq 0.2$:

$$\frac{P}{P_n/\Omega} + \frac{8}{9}\left(\frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega}\right) \leq 1.0 \quad \text{(ASD)}$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \quad \text{(LRFD)}$$

For $\frac{P_r}{P_c} < 0.2$:

$$\frac{P}{2P_n/\Omega} + \left(\frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega}\right) \leq 1.0 \quad \text{(ASD)}$$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \quad \text{(LRFD)}$$

where:

for compression	ϕ_c = 0.90 (LRFD)	Ω = 1.67 (ASD)
for bending	ϕ_b = 0.90 (LRFD)	Ω = 1.67 (ASD)

For a braced condition, the moment magnification factor B_1 is determined by $B_1 = \frac{C_m}{1-(P_u/P_{e1})} \leq 1.0$

where C_m is a modification factor accounting for end conditions

When not subject to transverse loading between supports in plane of bending:

= 0.6 – 0.4 (M_1/M_2) where M_1 and M_2 are the end moments and $M_1<M_2$. M_1/M_2 is positive when the member is bent in reverse curvature (same direction), negative when bent in single curvature.

When there is transverse loading between the two ends of a member:

= 0.85, members with restrained (fixed) ends

= 1.00, members with unrestrained ends

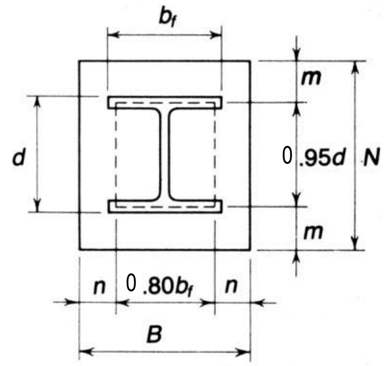
P_{e1} =Euler buckling strength

$$P_{e1} = \frac{\pi^2 EA}{\left(Kl/r\right)^2}$$

Criteria for Design of Connections and Tension Members

Refer to the specific note set.

Criteria for Design of Column Base Plates



Column base plates are designed for bearing on the concrete (concrete capacity) and flexure because the column "punches" down the plate and it could bend upward near the edges of the column (shown as $0.8b_f$ and $0.95d$). The plate dimensions are B and N and are preferably in full inches with thicknesses in multiples of 1/8 inches.

LRFD minimum thickness: $t_{min} = l\sqrt{\frac{2P_u}{0.9F_y BN}}$

where l is the larger of m, n and $\lambda n'$

$m = \frac{N - 0.95d}{2}$ $n = \frac{B - 0.8b_f}{2}$

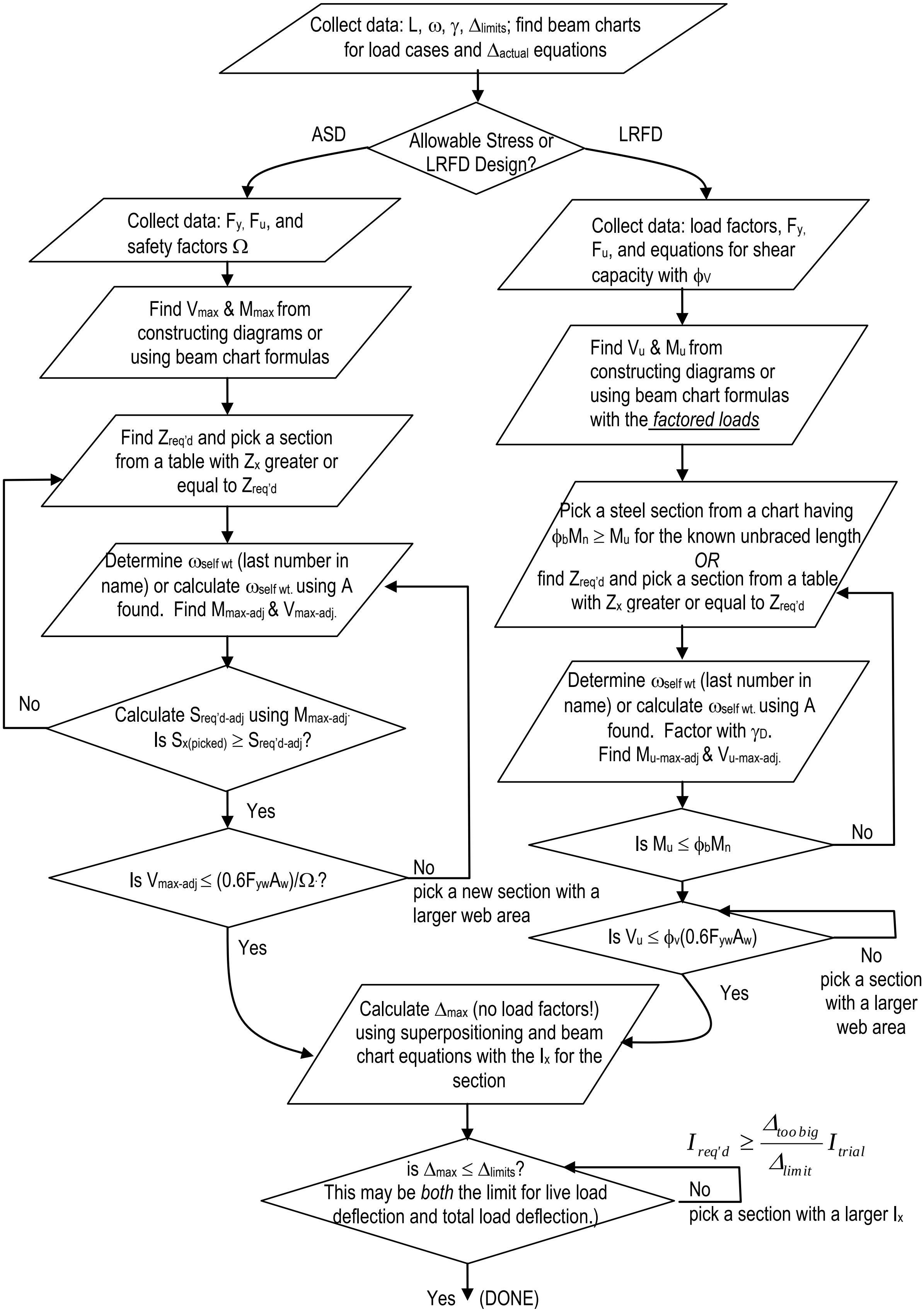
$n' = \frac{\sqrt{db_f}}{4}$ $\lambda = \frac{2\sqrt{X}}{(1+\sqrt{1-X})} \le 1$

where X depends on the concrete bearing capacity of $\phi_c P_p$, with

$\phi_c = 0.65$ and $P_p = 0.85f'_c A$

$$X = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c P_p} = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c(0.85f'_c)BN}$$

Beam Design Flow Chart



$$I_{req'd} \geq \frac{\Delta_{too\,big}}{\Delta_{limit}} I_{trial}$$

Listing of W shapes in Descending Order of Z_x for Beam Design

Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	I_x – SI (10^{6}mm.^{4})	Z_x – SI (10^{3}mm.3)
514	**7450**	**W33X141**	**3100**	**8420**
511	5680	W24X176	2360	8370
509	**7800**	**W36X135**	**3250**	**8340**
500	6680	W30X148	2780	8190
490	4330	W18X211	1800	8030
487	3400	W14X257	1420	7980
481	3110	W12X279	1290	7880
476	4730	W21X182	1970	7800
468	5170	W24X162	2150	7670
467	**6710**	**W33X130**	**2790**	**7650**
464	5660	W27X146	2360	7600
442	3870	W18X192	1610	7240
437	5770	W30X132	2400	7160
436	3010	W14X233	1250	7140
432	4280	W21X166	1780	7080
428	2720	W12X252	1130	7010
418	4580	W24X146	1910	6850
415	**5900**	**W33X118**	**2460**	**6800**
408	5360	W30X124	2230	6690
398	3450	W18X175	1440	6520
395	4760	W27X129	1980	6470
390	2660	W14X211	1110	6390
386	2420	W12X230	1010	6330
378	**4930**	**W30X116**	**2050**	**6190**
373	3630	W21X147	1510	6110
370	4020	W24X131	1670	6060
356	3060	W18X158	1270	5830
355	2400	W14X193	999	5820
348	2140	W12X210	891	5700
346	**4470**	**W30X108**	**1860**	**5670**
343	4080	W27X114	1700	5620
333	3220	W21X132	1340	5460
327	3540	W24X117	1470	5360
322	2750	W18X143	1140	5280
320	2140	W14X176	891	5240
312	**3990**	**W30X99**	**1660**	**5110**
311	1890	W12X190	787	5100
307	2960	W21X122	1230	5030
305	3620	W27X102	1510	5000
290	2460	W18X130	1020	4750
289	3100	W24X104	1290	4740
287	1900	W14X159	791	4700
283	**3610**	**W30X90**	**1500**	**4640**
280	3000	W24X103	1250	4590
279	2670	W21X111	1110	4570
278	3270	W27X94	1360	4560
275	1650	W12X170	687	4510
262	2190	W18X119	912	4290
260	1710	W14X145	712	4260
254	2700	W24X94	1120	4160
253	2420	W21X101	1010	4150
244	**2850**	**W27X84**	**1190**	**4000**
243	1430	W12X152	595	3980
234	1530	W14X132	637	3830
230	1910	W18X106	795	3770
224	**2370**	**W24X84**	**986**	**3670**
221	2070	W21X93	862	3620
214	1240	W12X136	516	3510
212	1380	W14X120	574	3470
211	1750	W18X97	728	3460
200	**2100**	**W24X76**	**874**	**3280**
198	1490	W16X100	620	3240
196	1830	W21X83	762	3210
192	1240	W14X109	516	3150
186	1530	W18X86	637	3050
185	1070	W12X120	445	3050
177	**1830**	**W24X68**	**762**	**2900**
174	1300	W16X89	541	2870
173	1110	W14X99	462	2830
169	1600	W21X73	666	2820
164	933	W12X106	388	2690
160	1330	W18X76	554	2670
159	**1480**	**W21X68**	**616**	**2620**
157	999	W14X90	416	2570
153	**1550**	**W24X62**	**645**	**2510**
147	1110	W16X77	462	2460
147	833	W12X96	347	2410
146	716	W10X112	298	2410
146	1170	W18X71	487	2390

(continued)

Listing of W Shapes in Descending order of Z_x for Beam Design (Continued)

Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	I_x – SI (10^6mm.^{4})	Z_x – SI (10^3mm.3)	Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	I_x – SI (10^6mm.^{4})	Z_x – SI (10^3mm.3)
144	**1330**	**W21X62**	**554**	**2360**	**66.5**	**510**	**W18X35**	**212**	**1090**
139	881	W14X82	367	2280	64.0	348	W12X45	145	1050
133	**1350**	**W24X55**	**562**	**2200**	63.5	448	W16X36	186	1050
132	1070	W18X65	445	2180	61.5	385	W14X38	160	1010
131	740	W12X87	308	2160	59.4	228	W8X58	94.9	980
130	954	W16X67	397	2130	57.0	307	W12X40	128	934
129	623	W10X100	259	2130	54.7	248	W10X45	103	900
129	1170	W21X57	487	2110	**54.5**	**340**	**W14X34**	**142**	**895**
126	**1140**	**W21X55**	**475**	**2060**	**53.7**	**375**	**W16X31**	**156**	**885**
126	795	W14X74	331	2060	51.2	285	W12X35	119	839
123	984	W18X60	410	2020	49.0	184	W8X48	76.6	803
118	662	W12X79	276	1950	**47.2**	**291**	**W14X30**	**121**	**775**
115	722	W14X68	301	1880	46.7	209	W10X39	87.0	767
113	534	W10X88	222	1850	**44.2**	**301**	**W16X26**	**125**	**724**
112	**890**	**W18X55**	**370**	**1840**	43.0	238	W12X30	99.1	706
110	**984**	**W21X50**	**410**	**1800**	**40.1**	**245**	**W14X26**	**102**	**659**
108	597	W12X72	248	1770	39.7	146	W8X40	60.8	652
107	**959**	**W21X48**	**399**	**1750**	38.5	171	W10X33	71.2	636
105	758	W16X57	316	1720	37.1	204	W12X26	84.9	610
102	640	W14X61	266	1670	36.6	170	W10X30	70.8	600
100	800	W18X50	333	1660	34.7	127	W8X35	52.9	569
96.8	455	W10X77	189	1600	**33.2**	**199**	**W14X22**	**82.8**	**544**
95.5	533	W12X65	222	1590	31.3	144	W10X26	59.9	513
95.4	**843**	**W21X44**	**351**	**1560**	30.4	110	W8X31	45.8	498
91.7	659	W16X50	274	1510	**29.2**	**156**	**W12X22**	**64.9**	**480**
90.6	712	W18X46	296	1490	27.1	98.0	W8X28	40.8	446
86.5	541	W14X53	225	1430	**26.0**	**118**	**W10X22**	**49.1**	**426**
86.4	475	W12X58	198	1420	**24.6**	**130**	**W12X19**	**54.1**	**405**
85.2	394	W10X68	164	1400	23.1	82.7	W8X24	34.4	379
82.1	586	W16X45	244	1350	**21.4**	**96.3**	**W10X19**	**40.1**	**354**
78.4	**612**	**W18X40**	**255**	**1280**	20.4	75.3	W8X21	31.3	334
78.1	484	W14X48	201	1280	**20.1**	**103**	**W12x16**	**42.9**	**329**
77.3	425	W12X53	177	1280	18.6	81.9	W10X17	34.1	306
74.4	341	W10X60	142	1220	**17.3**	**88.6**	**W12X14**	**36.9**	**285**
72.2	**518**	**W16X40**	**216**	**1200**	17.0	61.9	W8X18	25.8	279
71.8	391	W12X50	163	1180	15.9	68.9	W10X15	28.7	262
69.6	272	W8X67	113	1150	13.6	48.0	W8X15	20.0	223
69.4	428	W14X43	178	1140	**12.6**	**53.8**	**W10X12**	**22.4**	**206**
66.5	303	W10X54	126	1090	11.4	39.6	W8X13	16.5	187
					8.87	**30.8**	**W8X10**	**12.8**	**145**

Available Critical Stress, $\phi_c F_{cr}$, for Compression Members, ksi (F_y = *36* ksi and ϕ_c = 0.90)

KL/r	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$
1	32.4	41	29.7	81	22.9	121	15.0	161	8.72
2	32.4	42	29.5	82	22.7	122	14.8	162	8.61
3	32.4	43	29.4	83	22.5	123	14.6	163	8.50
4	32.4	44	29.3	84	22.3	124	14.4	164	8.40
5	32.4	45	29.1	85	22.1	125	14.2	165	8.30
6	32.3	46	29.0	86	22.0	126	14.0	166	8.20
7	32.3	47	28.8	87	21.8	127	13.9	167	8.10
8	32.3	48	28.7	88	21.6	128	13.7	168	8.00
9	32.3	49	28.6	89	21.4	129	13.5	169	7.91
10	32.2	50	28.4	90	21.2	130	13.3	170	7.82
11	32.2	51	28.3	91	21.0	131	13.1	171	7.73
12	32.2	52	28.1	92	20.8	132	12.9	172	7.64
13	32.1	53	27.9	93	20.5	133	12.8	173	7.55
14	32.1	54	27.8	94	20.3	134	12.6	174	7.46
15	32.0	55	27.6	95	20.1	135	12.4	175	7.38
16	32.0	56	27.5	96	19.9	136	12.2	176	7.29
17	31.9	57	27.3	97	19.7	137	12.0	177	7.21
18	31.9	58	27.1	98	19.5	138	11.9	178	7.13
19	31.8	59	27.0	99	19.3	139	11.7	179	7.05
20	31.7	60	26.8	100	19.1	140	11.5	180	6.97
21	31.7	61	26.6	101	18.9	141	11.4	181	6.90
22	31.6	62	26.5	102	18.7	142	11.2	182	6.82
23	31.5	63	26.3	103	18.5	143	11.0	183	6.75
24	31.4	64	26.1	104	18.3	144	10.9	184	6.67
25	31.4	65	25.9	105	18.1	145	10.7	185	6.60
26	31.3	66	25.8	106	17.9	146	10.6	186	6.53
27	31.2	67	25.6	107	17.7	147	10.5	187	6.46
28	31.1	68	25.4	108	17.5	148	10.3	188	6.39
29	31.0	69	25.2	109	17.3	149	10.2	189	6.32
30	30.9	70	25.0	110	17.1	150	10.0	190	6.26
31	30.8	71	24.8	111	16.9	151	9.91	191	6.19
32	30.7	72	24.7	112	16.7	152	9.78	192	6.13
33	30.6	73	24.5	113	16.5	153	9.65	193	6.06
34	30.5	74	24.3	114	16.3	154	9.53	194	6.00
35	30.4	75	24.1	115	16.2	155	9.40	195	5.94
36	30.3	76	23.9	116	16.0	156	9.28	196	5.88
37	30.1	77	23.7	117	15.8	157	9.17	197	5.82
38	30.0	78	23.5	118	15.6	158	9.05	198	5.76
39	29.9	79	23.3	119	15.4	159	8.94	199	5.70
40	29.8	80	23.1	120	15.2	160	8.82	200	5.65

Available Critical Stress, $\phi_c F_{cr}$, for Compression Members, ksi (F_y = 50 ksi and ϕ_c = 0.90)

KL/r	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$	*KL/r*	$\phi_c F_{cr}$
1	45.0	41	39.8	81	27.9	121	15.4	161	8.72
2	45.0	42	39.6	82	27.5	122	15.2	162	8.61
3	45.0	43	39.3	83	27.2	123	14.9	163	8.50
4	44.9	44	39.1	84	26.9	124	14.7	164	8.40
5	44.9	45	38.8	85	26.5	125	14.5	165	8.30
6	44.9	46	38.5	86	26.2	126	14.2	166	8.20
7	44.8	47	38.3	87	25.9	127	14.0	167	8.10
8	44.8	48	38.0	88	25.5	128	13.8	168	8.00
9	44.7	49	37.8	89	25.2	129	13.6	169	7.91
10	44.7	50	37.5	90	24.9	130	13.4	170	7.82
11	44.6	51	37.2	91	24.6	131	13.2	171	7.73
12	44.5	52	36.9	92	24.2	132	13.0	172	7.64
13	44.4	53	36.6	93	23.9	133	12.8	173	7.55
14	44.4	54	36.4	94	23.6	134	12.6	174	7.46
15	44.3	55	36.1	95	23.3	135	12.4	175	7.38
16	44.2	56	35.8	96	22.9	136	12.2	176	7.29
17	44.1	57	35.5	97	22.6	137	12.0	177	7.21
18	43.9	58	35.2	98	22.3	138	11.9	178	7.13
19	43.8	59	34.9	99	22.0	139	11.7	179	7.05
20	43.7	60	34.6	100	21.7	140	11.5	180	6.97
21	43.6	61	34.3	101	21.3	141	11.4	181	6.90
22	43.4	62	34.0	102	21.0	142	11.2	182	6.82
23	43.3	63	33.7	103	20.7	143	11.0	183	6.75
24	43.1	64	33.4	104	20.4	144	10.9	184	6.67
25	43.0	65	33.0	105	20.1	145	10.7	185	6.60
26	42.8	66	32.7	106	19.8	146	10.6	186	6.53
27	42.7	67	32.4	107	19.5	147	10.5	187	6.46
28	42.5	68	32.1	108	19.2	148	10.3	188	6.39
29	42.3	69	31.8	109	18.9	149	10.2	189	6.32
30	42.1	70	31.4	110	18.6	150	10.0	190	6.26
31	41.9	71	31.1	111	18.3	151	9.91	191	6.19
32	41.8	72	30.8	112	18.0	152	9.78	192	6.13
33	41.6	73	30.5	113	17.7	153	9.65	193	6.06
34	41.4	74	30.2	114	17.4	154	9.53	194	6.00
35	41.1	75	29.8	115	17.1	155	9.40	195	5.94
36	40.9	76	29.5	116	16.8	156	9.28	196	5.88
37	40.7	77	29.2	117	16.5	157	9.17	197	5.82
38	40.5	78	28.8	118	16.2	158	9.05	198	5.76
39	40.3	79	28.5	119	16.0	159	8.94	199	5.70
40	40.0	80	28.2	120	15.7	160	8.82	200	5.65